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Engineering Assessment of Flooding at Arroyo Doble Subdivision

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Basis of the Analyses, Opinions and Conclusions

The analysis, opinions and conclusions expressed in this report are based upon information available to the authors at the time of the writing of the report. If additional or updated information is presented, the authors reserve the right to examine the information and reform our analyses and resulting opinions in response to the information. The opinions in this report are based on the degree of care and skill ordinarily exercised under similar conditions by reputable members of the civil engineering profession practicing in the same or similar service.

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I. Executive Summary

This analysis has found that the embankment failure of the UPRR Railroad embankment in October of 2015 may have produced up to 7,169 cubic feet per second (cfs) of a flood surge that flowed into the Arroyo Doble subdivision. If the embankment had not failed but instead, 1) overtopped and not breached, or 2) if it had a culvert placed to ease the pressure of flow, the maximum released flow for this event would have been between 900 and 1,300 cfs, depending on the storm distribution. Under these two scenarios, the number of houses inundated would have been fewer and the extent of inundated would have been less. Had the UPRR Railroad corrected this drainage deficiency, the amount of damage caused by this failure could have been significantly diminished.

It does appear that there was a major failure in the past at this location. Based on review of aerial imagery, it appears that this location has been jeopardized and damaged from a previous rainfall event. In October 30 of 2013, there was an approximately 20-year rainfall event in the same drainage basin. Based on review of Google Earth imagery, it appears that a breach at the same location had occurred and was repaired by the UPRR staff.

According to the UPRR Engineering Track Maintenance Handbook (Revised in 2015) such a failure should have been reported and remedied, and a culvert or some drainage design should have been initiated immediately after the 2013 flood. Had this been done, it is possible that the 2015 flood event would not have produced such a large flood surge from the breached embankment and the damages to the neighborhood could have been significantly diminished.

II. Introduction

On October 30th of 2015, a large storm event produced heavy rainfall over the Arroyo Doble subdivision and surrounding watersheds. Runoff from this storm event caused ponding upstream of the Union Pacific Railroad (UPRR) embankment near the subdivision. The ponded water eventually overcame the embankment's stability and breached the railroad embankment, sending copious amounts of water into the Arroyo Doble subdivision. This breach is shown in Figure 1. The water flooded numerous houses and caused extensive damage as the flood water from the breach traveled through the subdivision and eventually exiting to Onion Creek further downstream.

Figure 1 - Picture of the breached railroad embankment provided by a resident of the Arroyo Doble neighborhood

This analysis was requested as part of a lawsuit against the UPRR Company (Gallo et al v. UPRR, Case No. 1:17-CV-00854-RP).

This analysis includes:

1. Performing a hydrologic and hydraulic analysis to re-create the flood event of October 30 – 31, 2015.
2. Assessing measured high-water marks at and near the residential structures.
3. Reviewing UPRR maintenance records, aerial imagery, and any other history of previous drainage issues at and near the breach location.
4. Reviewing maintenance standards for railroads in Texas.
5. Comparing the impacts of 1) if the UPRR embankment was <u>not in place</u>, 2) with the embankment <u>in place</u> but the breach had not occurred, and 3) with the embankment in place and the breach occurs.

In the following, the reader will find a discussion of the means and methods of our analysis and how any conclusions presented in this report were developed.

III. Hydrologic and Hydraulic Analysis

<u>Hydrology</u>

This analysis developed a hydrologic model using the U.S. Army Corps of Engineers Hydrologic Modeling System (HEC-HMS), version 4.2.1. Within the HEC-HMS model, sub-basin delineation, watershed parameters, channel parameters, and breach parameters were used to estimate the flow rate and amount of water that was released from the Union Pacific Railroad (UPRR) embankment that breached on October of 2015. Hydrology, in the context of this report, consists of identifying spatial and temporal distributions of rainfall intensities and then using a series of engineering methods to transform that rainfall into flood runoff. Once the rainfall is transformed into runoff, it is then routed through the watershed using equations for hydraulic routing.

Rainfall Depths

The rainfall depth and distribution estimates were collected from four (4) sites within the surrounding area of interest. Information was collected from the National Oceanic and Atmospheric Administration (NOAA)[1]. The nearest rainfall gage site is the San Leanna Gage, which is located approximately 1.4 miles from the centroid of the watershed. Daily rainfall depths for October 2015 were collected for the following sites:

1.) CREEDMOOR 1.5 NNW, TX US US1TXTV0135
2.) MANCHACA 2.1 ENE, TX US US1TXHYS028
3.) SAN LEANNA 0.1 SSE, TX US US1TXTV0175
4.) BUDA 0.7 SW, TX US US1TXHYS131

The 24-hour, daily depths from the four sites were compared using the inverse distance weighted technique (IDW) to estimate the total rainfall depth over the centroid of the watershed upstream of the Arroyo Doble subdivision. This resulted in a total rainfall depth of ***12.10 inches over 24 hours***. Table 1 presents the rainfall amount, distances and results from the IDW technique. Figure 2 is an image of the gage locations placed into Google Earth.

Table 1 – Gage Depth and Distance from Centroid of Drainage Area, October 31, 2015

Gage ID	Distance from Drainage Area (feet)	1/D^2	24 Hour Rainfall Depth (inch)
Manchaca	20150	2.46292E-09	9.33
San Leanna	7641	1.71277E-08	12.62
Buda	18811	2.82603E-09	11.98
Creedmore	24457	1.67184E-09	11.00
	Sum	2.40885E-08	**12.10**

[1] https://www.ncdc.noaa.gov/data-access/land-based-station-data

Figure 2 - Google Earth Image of Rainfall Locations in Respect to Drainage Area Centroid

From the Austin Drainage Criteria Manual[2], Section 2.4.3, a 24 hour rainfall depth of 12.0 inches correlates to a 1 in 250 year storm, or a 0.4% chance of this storm occurring in any given year.

Rainfall Distributions

Rainfall data for this storm event did not include temporal distributions (depth of rainfall over time) for any period less than 24 hours. To translate the 24 hour rainfall depths into a time series rainfall pattern, the synthetic rainfall distribution described by the Soils Conservation Service (SCS) Type III rainfall distribution was applied for this study. This distribution was

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[2] https://library.municode.com/tx/austin/codes/drainage_criteria_manual?nodeId=S2DESTRU_2.1.0GE

selected because it is recommended by the City of Austin's Drainage Criteria Manual (Drainage Manual)[3].

Within the Drainage Manual, there are no recommendations on the total time duration of a storm that should be used for this type of analysis except that the storm duration for a watershed should be equal to or longer than the time of concentration of that watershed. Following this rule, and the recommendations within the Drainage Manual, the storm duration would need to be longer than 1.4 hours. The analysis investigated storm durations of 6, 12, and 24 hours for the October 2015 event. These durations are longer than 1.4 hours, thereby meeting the criteria. Figure 3 below presents the storm precipitation distributions for the three storm durations that were used in this analysis.

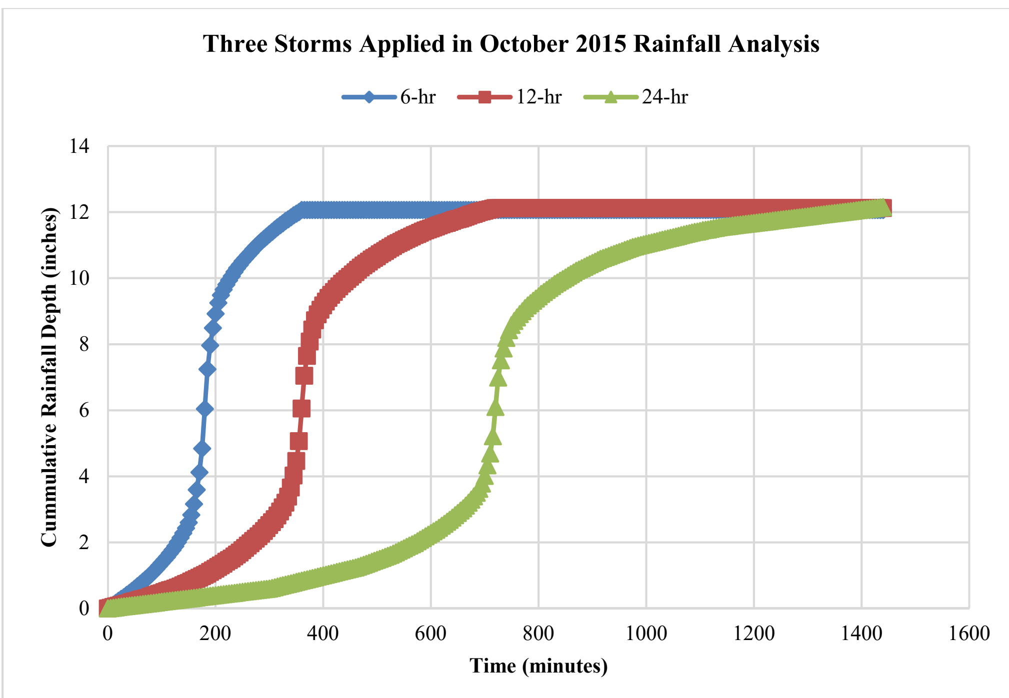

Figure 3 - Three Storm Distributions Tested in this Analysis

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[3] https://library.municode.com/TX/Austin/codes/Drainage_Criteria_Manual?nodeId=S2DESTRU)

Watershed Delineation and Runoff

Drainage basins were delineated within the Watershed Modeling System (WMS V. 10.1)[4]. Within WMS, contributing drainage areas, drainage slopes, and additional watershed parameters were computed. Hydrologic parameters, such as soil textures, depression losses, and basin lag times were estimated using recommended procedures within TR-55[5]. Hydrologic Soil Groupings were downloaded from the US Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS)[6] and geo-spatially overlain to the drainage areas. Land use data was taken from the National Land Cover Database (NLCD). Weighted Curve Numbers (CN) values, initial abstraction values, and basin lag times were computed for each sub basin drainage area. The Figures 4 and 5 and Table 2 present the watershed delineation, HEC-HMS Model set up, and the watershed parameters, respectively.

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[4] https://www.aquaveo.com/downloads-wms

[5] https://www.nrcs.usda.gov/Internet/FSE_DOCUMENTS/stelprdb1044171.pdf

[6] https://www.nrcs.usda.gov/wps/portal/nrcs/detail/soils/survey/geo/?cid=nrcs142p2_053629

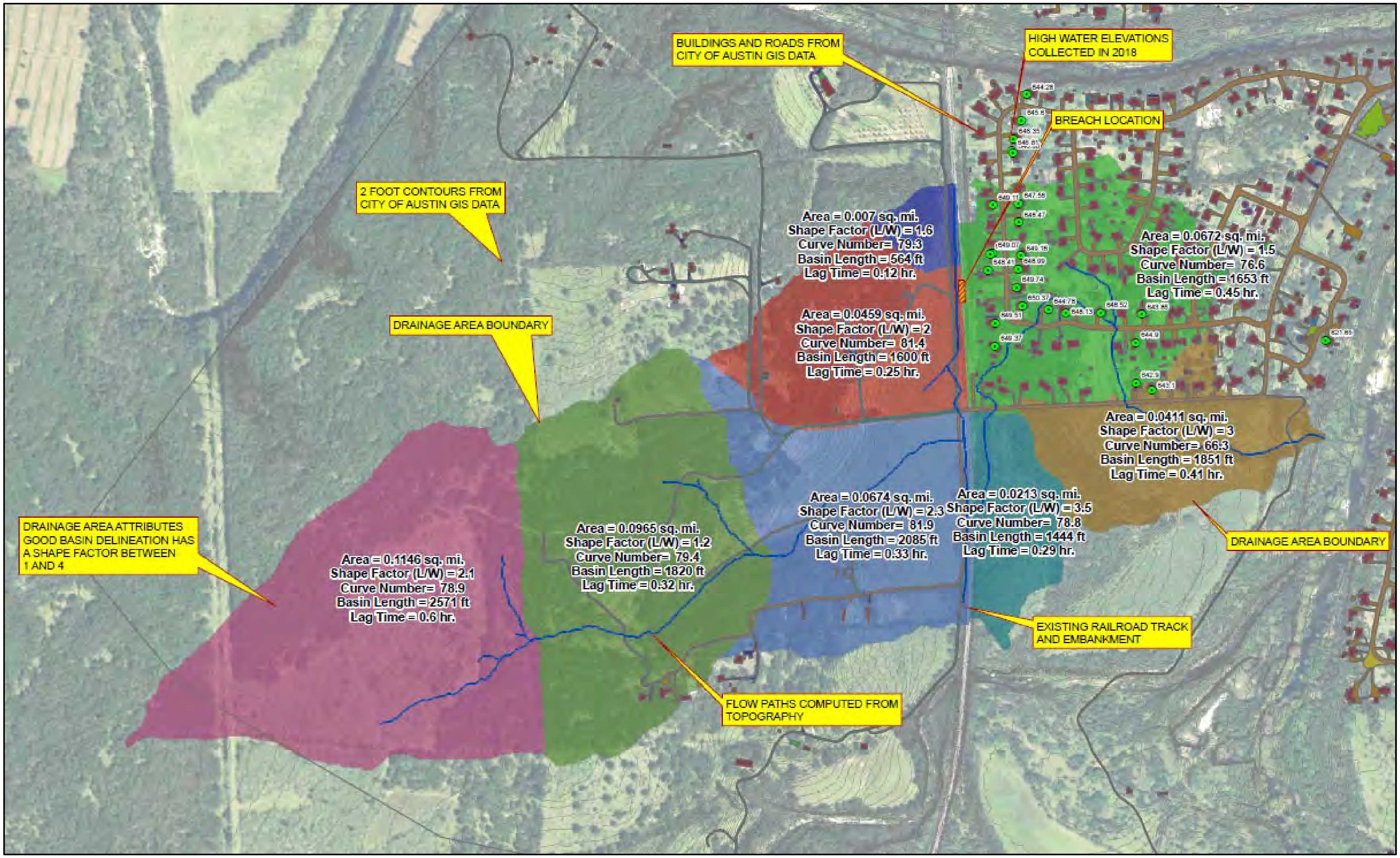

Figure 4 – Image of the Watershed Delineation and Basin Parameters

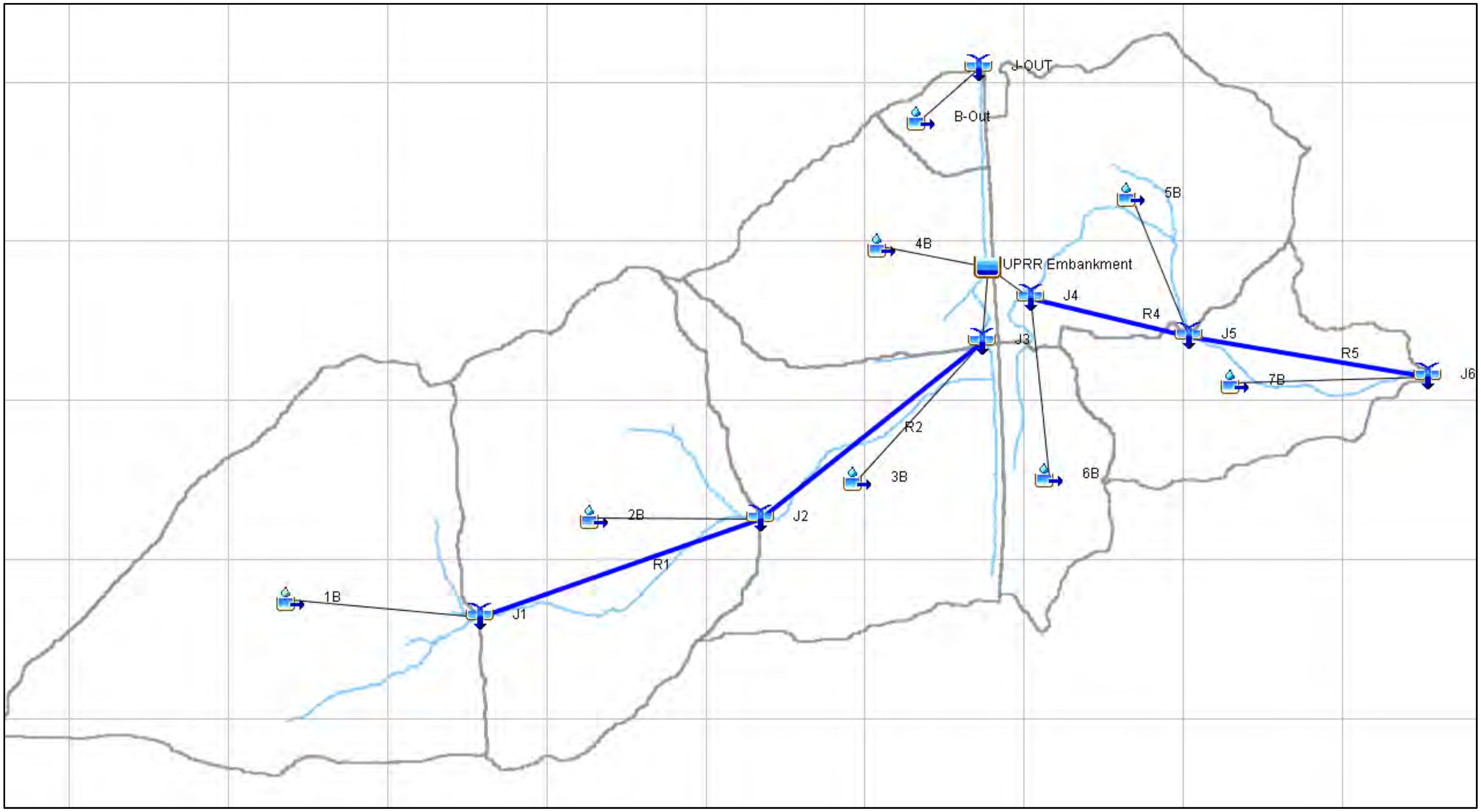

Figure 5 - Image Showing HEC-HMS Model Setup and Connectivity

Table 2 - HEC-HMS Model Basin Parameters

Basin ID	Initial Abstraction (inch)	Curve Number	Lag Time (min)	Description
B-out	0.52	79.29	7.40	Drains north, does not contribute to breach analysis
1B	0.53	78.92	36.26	Upstream of UPRR Embankment
2B	0.52	79.39	19.32	Upstream of UPRR Embankment
3B	0.44	81.88	19.70	Upstream of UPRR Embankment
4B	0.46	81.38	15.11	Upstream of UPRR Embankment, Drains Directly to Embankment
5B	0.61	76.55	27.18	Downstream of UPRR Embankment
6B	0.54	78.77	17.24	Downstream of UPRR Embankment
7B	1.02	66.31	24.89	Downstream of UPRR Embankment

Runoff from each sub-basin was routed to the next downslope sub-basin using the Muskingum-Cunge method. A representative eight-point cross section was cut from the Digital Elevation Model (DEM) surface and input into the paired data manager within HEC-HMS.

Breach Analysis

A breach analysis was developed within HEC-HMS to estimate the rate and amount of water that was pushed through the railroad track and embankment for the October 2015 event. Parameters for the breach analysis were developed from the survey information collected in April – May of 2018 by Mr. Bill O'Hara (RPLS, LSLS), the information from UPRR responses on breach location and widths, and the photographs provided by neighborhood residents (see Figure 6).

The main breach was estimated to be 100 feet long and assumed a scour hole approximately 6 feet deep. According to UPRR responses to questions (DEFENDANT'S OBJECTIONS AND ANSWERS TO PLAINTIFFS' INTERROGATORIES, dated July 6, 2018),

there was a second, smaller breach of about 25 feet in width. This would make the total breach length, approximately 125 feet. Alternatively, it could be assumed that the smaller breach was less consequential (essentially bank sloughing) and that the large breach of 100 feet is the dominant breach mechanism. The assumption was made that the effective breach was ultimately 100 feet long and 6 feet deep.

Figure 6 - Image of Breach Location taken after the 2015 Event

This analysis tested two types of breach failures, overtopping and piping. Piping is considered breach of the embankment under the rails. Breach parameters were taken from the HEC RAS Technical Reference Manual, Tables 14.3 and 14.2, resulting in the overflow coefficient of 2.6 and a piping coefficient of 0.60 to be used. Failure times and side slopes were estimated from Table 14.3 and a value of 0.10 hours of failure time was applied. Elevations to define the breach height were developed using the April – May 2018 survey data provided by Mr. Bill O'Hara (RPLS, LSLS) and the photographs provided by neighborhood residents (see

Figure 6 as an example). The Figure 7 is a railroad embankment cross section developed from the 2018 survey and the Austin GIS topography. The figure calls out the assumed parameters for the breach analysis.

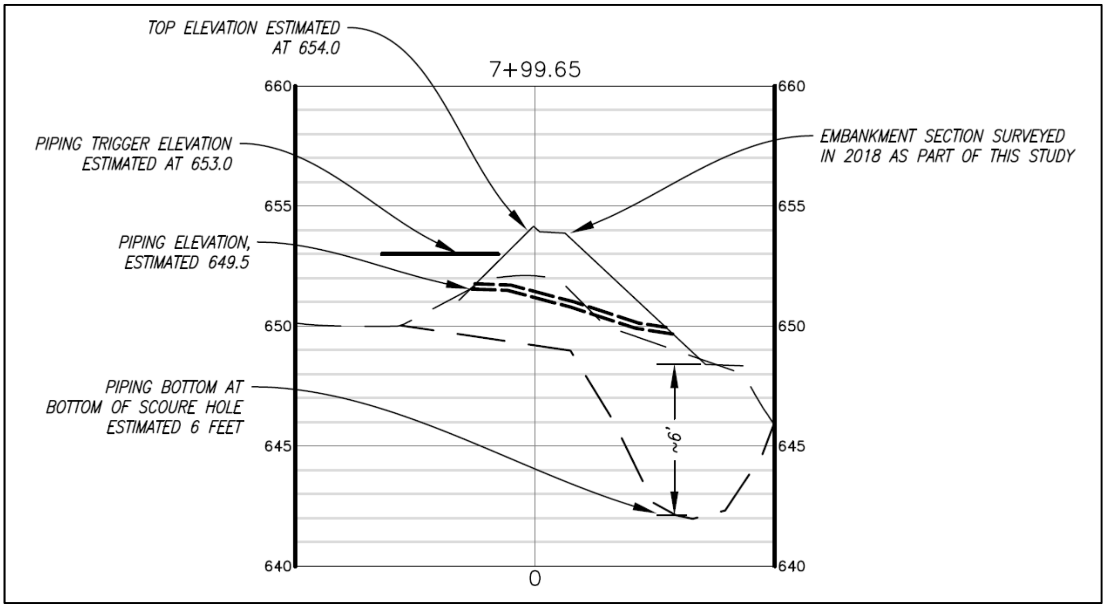

Figure 7 - Cross Section of UPRR Embankment and Breach Parameters

Figure 8 presents a visual representation of the two types of breaches tested in this analysis.

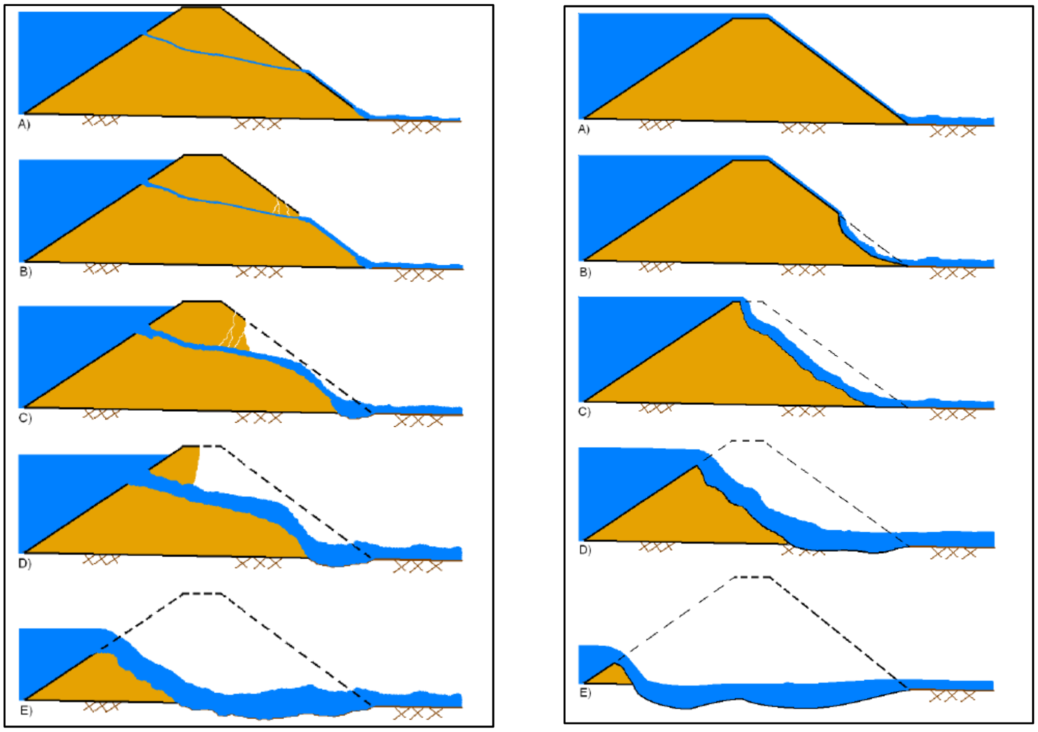

Figure 8 - Example of Piping (left) and Overtopping (Right) Failures taken from the HEC RAS Hydraulic Reference Manual

Using the three storm distributions/durations (6, 12 and 24 hours), two breach scenarios (piping and overtopping), and two types of breach progressions (linear and Sine wave type), 12 breach discharges were computed within the HEC-HMS model. For these 12 scenarios, the maximum discharge was 3,996 cfs and the lowest estimated discharge was 1,120 cfs. These results are summarized below in Table 3.

Table 3 – Results from Breach Analysis using Three Storm Distributions and Four Breach Conditions

Storm Type	Breach Type	Peak Discharge (cfs)
6 Hour	Piping Linear	2,446
	Piping S Curve	3,493
	Overtopping Linear	2,214
	Overtopping S Curve	3,996
12 Hour	Piping Linear	1,173
	Piping S Curve	3,686
	Overtopping Linear	1,127
	Overtopping S Curve	1,120
24 Hour	Piping Linear	2,050
	Piping S Curve	2,975
	Overtopping Linear	1,519
	Overtopping S Curve	2,904
	Max	**3,996**
	Min	**1,120**
	Average	**2,392**

An additional breach progression was tested by adjusting the breach triggering method to be triggered at a specific time instead of at a specific elevation. This method was tested on the 24 hour and 6 hour piping scenario with the Sine Wave progression method. It was found that this combination produced the largest breach analysis peak flow of 4,637 cfs for the 24 hour storm and 7,169 for the 6 hour storm. The two breach hydrographs are presented below in Figures 9 and 10.

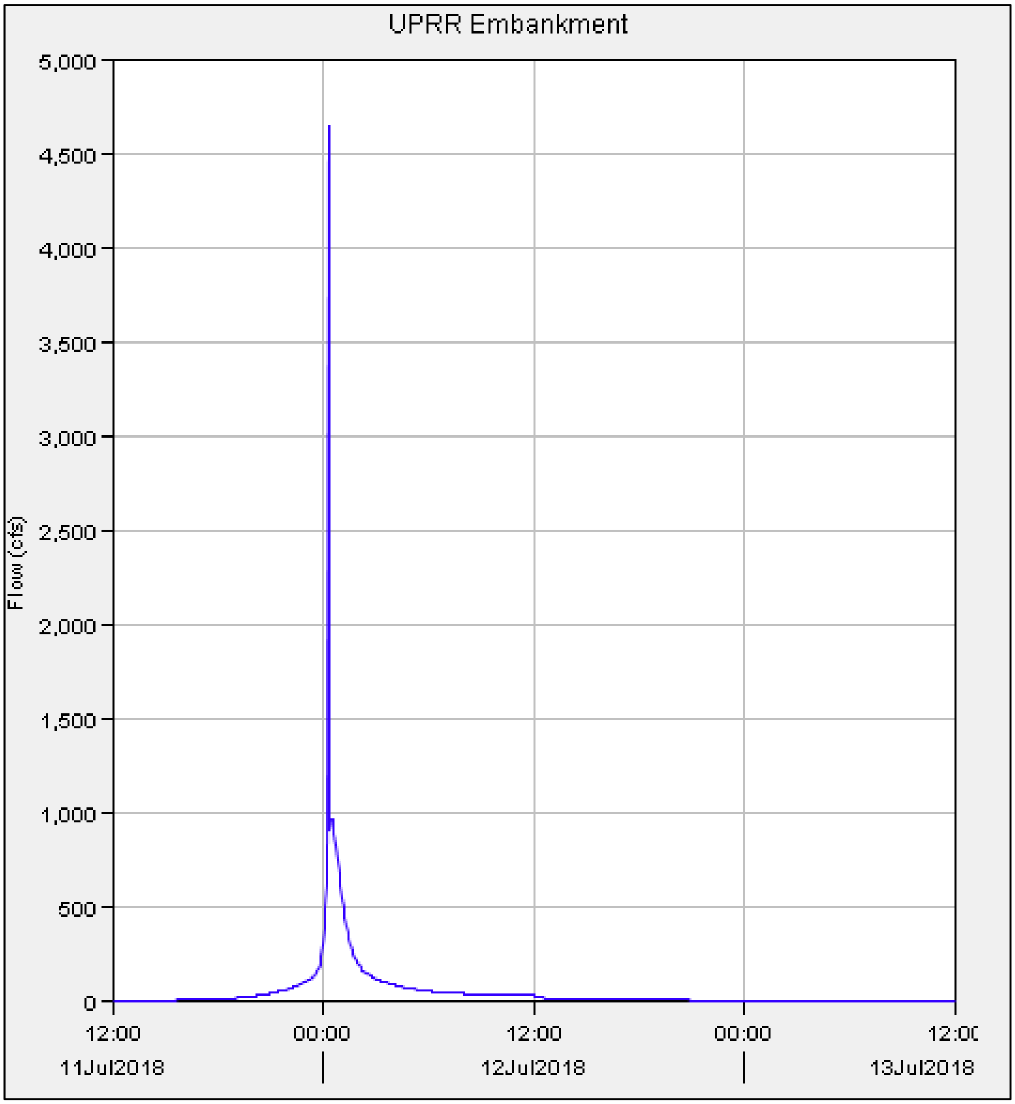

Figure 9 - Maximum Breach Hydrograph from a Piping Failure, with Time Triggered Beach, and a Sine Wave Progression under the 24-Hour Storm

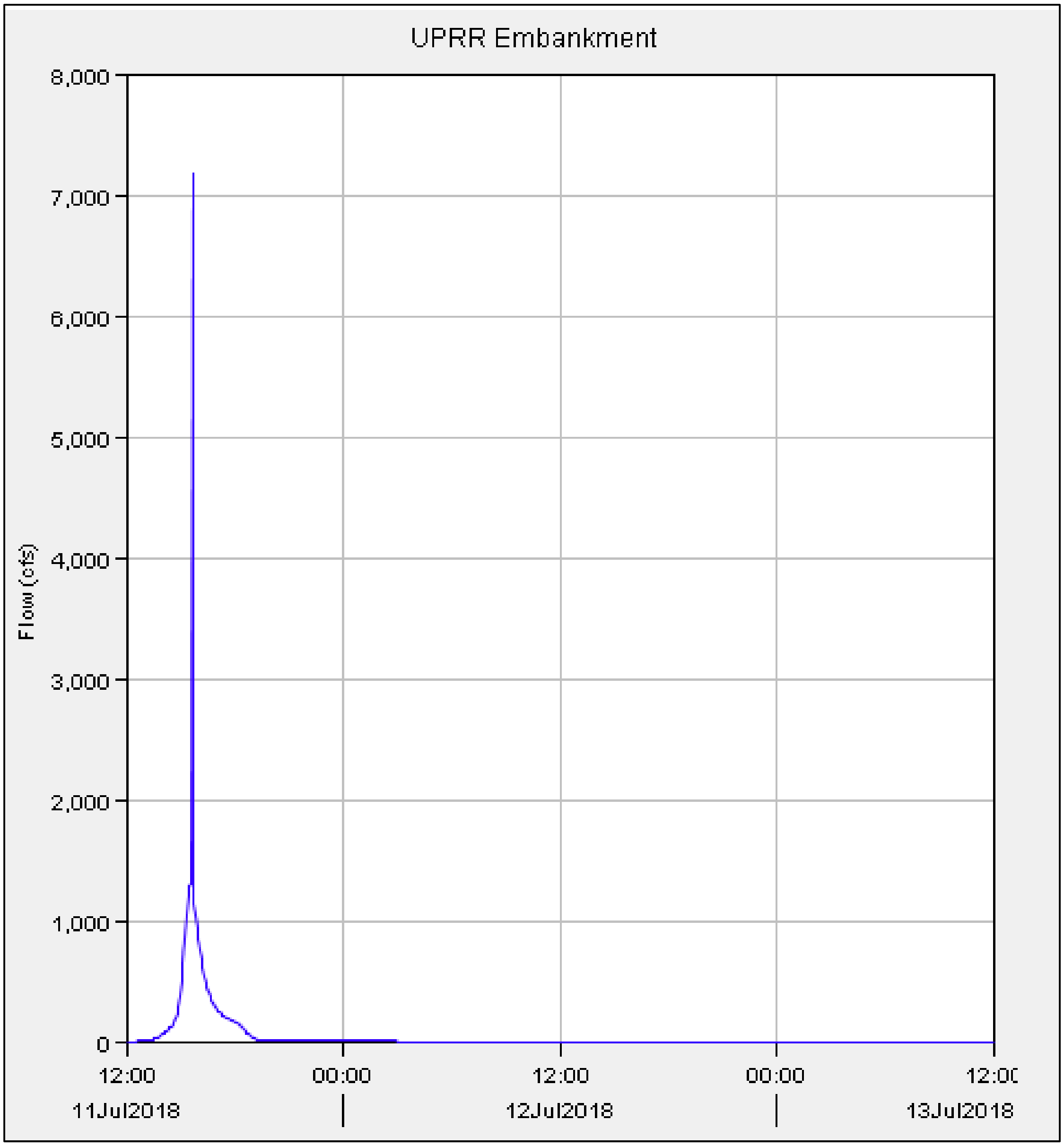

Figure 10 - Maximum Breach Hydrograph from a Piping Failure, with Time Triggered Beach, and a Sine Wave Progression under the 6 Hour Storm

<u>Hydraulic Analysis</u>

Due to time constraints, not all scenarios in Table 3 were analyzed hydraulically to determine the water depths in the Arroyo Doble subdivision. Instead, the following scenarios were hydraulically analyzed:

1. Steady Flow (constant peak flow) Breach Scenario of 3,996 cfs (the maximum discharge from Table 3)
2. Steady Flow (constant peak flow) Breach Scenario of 2,390 cfs (the average discharge from Table 3).
3. Unsteady Flow Hydrograph (flow varies with time) with a modified surface that eliminated the UPRR Railroad Track (Railroad embankment does not exist and therefore no breach occurred).
4. Unsteady Flow Hydrograph (flow varies with time) into UPRR Embankment with no breach (Railroad embankment is overtopped but does not fail).
5. Unsteady Flow Breach Hydrograph (flow varies with time) from Figure 9 above (Railroad embankment fails) (24-hour storm).
6. Unsteady Flow Breach Hydrograph (flow varies with time) from Figure 10 above (Railroad embankment fails) (6-hour storm).

An HEC RAS 2D model was used and each analysis utilized the full momentum equation within the hydraulic analysis. Results from the 2D Hydraulic Model produce depth, velocity, and water surface elevation grids. These grids have multiple values of depth and velocity surrounding each building. To analyze the many data points, the grid values were spatially joined with points around and inside the buildings. Then a maximum energy value was taken from the surrounding points. Figure 11 illustrates this methodology.

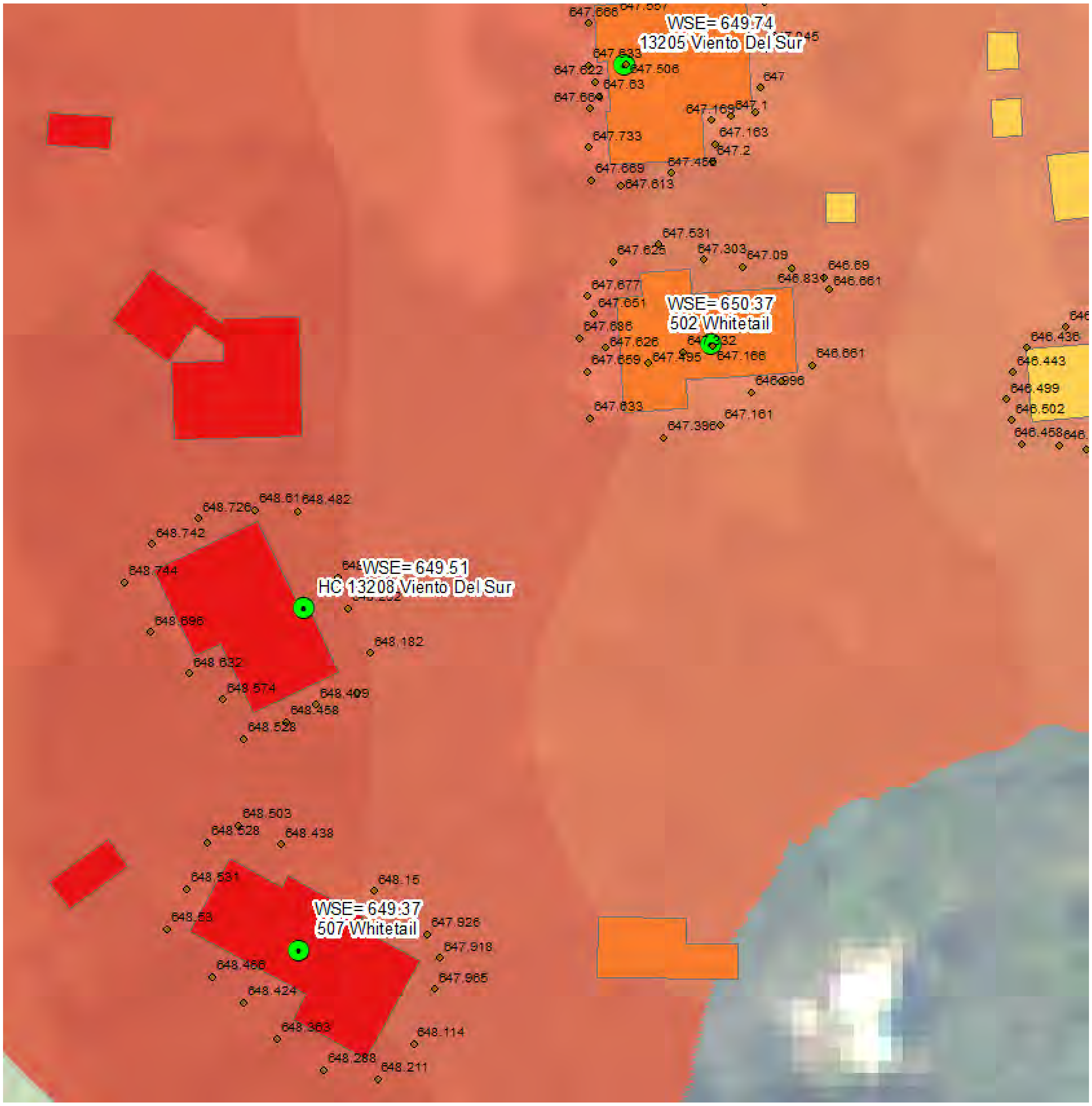

Figure 11 - Image of the WSE Grid and Energy Values Surrounding Structures, showing that the Max Energy is usually the Approach Energy on the Structure that is translated into a Pounding Wave that runs ups the outside of the Structure

The hydraulic model developed for this work incorporated 2-foot contours and planimetric data gathered during the data collection. Planimetric data were separated into distinct categories for appropriate resolution and refinement within the HEC RAS 2D model. These categories are taken from the GIS data and merged into one layer then a roughness value

assigned within the HEC RAS model. Roughness[7] values for each category are presented in Table 4.

Table 4 – Roughness Values

Category from GIS Data	Applied Roughness Value
paved parking	0.035
recreation court/ball field	0.045
remaining pervious area	0.065
Sidewalk	0.035
Structure	0.099
Tank	0.099
Uncovered	0.045
Unpaved	0.055
unpaved athletic field	0.055
unpaved parking	0.055
Bridge	0.065
edge of paved road	0.045
edge of pavement	0.045
edge of unpaved road	0.045
in ground	0.035
Misc	0.055
Patio	0.035
Paved	0.035

It is important to note that more refinement[7] of these hydraulic models and input parameters may be based on additional information with more detailed data collection efforts. Such detailed data may provide better calibration points to refine the hydraulic model. Tables 5, 6, and 7 present the results of the analyses

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[7] Roughness values are a common method applied to estimate friction losses within open channel hydraulics. Estimates for this study were taken from CHP. 12 of the HEC RAS Technical Manual (http://www.hec.usace.army.mil/software/hec-ras/documentation/HEC-RAS%205.0%20Reference%20Manual.pdf)

Table 5 - Results of the Steady Flow 2D Analysis for Railroad Embankment Failure

Address	Surveyed High Water Mark	Steady State Model Q=2,390 cfs			Steady State Model Q=3,996 cfs			Difference (Δ) in Surveyed HWM to Total Energy	
		WSE (ft)	Velocity (ft/s)	Total Energy (ft)	WSE (ft)	Velocity (ft/s)	Total Energy (ft)	Q=2,390	Q=3,996
HC 600 Bear Canyon	644.28	644.08	0.22	644.08	644.30	0.95	644.31	-0.20	0.03
Watermark 601 Bear Canyon	645.60	644.43	0.31	644.43	644.82	0.79	644.83	-1.17	-0.77
13003 Viento Del Sur	646.35	644.96	0.37	644.97	645.27	0.95	645.29	-1.38	-1.06
HC 13101 Viento Del Sur	646.81	646.01	0.12	646.01	646.35	0.44	646.35	-0.80	-0.46
13106 Viento Del Sur	649.11				646.72	0.21	646.72		-2.39
HC 13107 Viento Del Sur	647.56	645.97	0.43	645.97	646.60	0.82	646.61	-1.59	-0.95
13109 Viento Del Sur	648.47	646.22	1.12	646.24	646.81	1.56	646.85	-2.23	-1.62
13202 Viento Del Sur	648.41	647.38	4.27	647.66	647.92	5.28	648.36	-0.75	-0.05
13202 Viento Del Sur	648.41	648.00	4.00	648.25	648.67	4.96	649.05	-0.16	0.64
HC 13203 Viento Del Sur	648.99	646.69	3.30	646.86	647.29	3.86	647.52	-2.13	-1.47
13205 Viento Del Sur	649.74	647.01	2.20	647.08	647.57	3.14	647.72	-2.66	-2.02
502 Whitetail	650.37	647.07	1.60	647.11	647.60	2.27	647.68	-3.26	-2.69
HC 13208 Viento Del Sur	649.51	648.13	1.02	648.15	648.66	1.80	648.71	-1.36	-0.80
507 Whitetail	649.37	648.13	0.10	648.13	648.52	0.64	648.53	-1.24	-0.84
500 Whitetail	644.78	645.66	1.46	645.69	646.38	2.20	646.46	0.91	1.68
Front door 412 Whitetail	648.13	645.13	1.91	645.19	645.83	2.81	645.96	-2.94	-2.17
408 Whitetail	646.52	643.55	3.00	643.69	644.29	3.42	644.47	-2.83	-2.05
404 Whitetail	643.85	643.18	0.51	643.18	643.94	1.30	643.96	-0.67	0.11
407 Whitetail	644.90	642.29	2.33	642.38	643.12	3.06	643.27	-2.52	-1.63
314 Horsethief Tr.	642.90	640.74	3.53	640.94	641.48	4.37	641.78	-1.96	-1.12
Front door 312 Horsethief Tr.	643.10	640.22	2.44	640.31	640.91	3.52	641.10	-2.79	-2.00
							Average	**-1.59**	**-1.03**

*Computational Cells with no data did not show structure being inundated in hydraulic scenario

Table 6 - Results of the Unsteady Flow 2D Analysis

		Existing No Breach			If Track was Removed			Breach with Qp=4,637			Breach with Qp=7,169			Δ Surveyed HWM to Total Energy			
Address	Surveyed High Water Mark (HWM)	WSE (ft)	Velocity (ft/s)	Total Energy (ft)	WSE (ft)	Velocity (ft/s)	Total Energy (ft)	WSE (ft)	Velocity (ft/s)	Total Energy (ft)	WSE (ft)	Velocity (ft/s)	Total Energy (ft)	With Embankment Overtopping Only	If no Embankment Existed	Overtopping and Breach (Qp=4,637)	Overtopping and Breach (Qp=7,169)
HC 600 Bear Canyon	644.28							644.05	0.22	644.05	644.34	1.14	644.36			-0.23	0.08
Watermark 601 Bear Canyon	645.60							644.46	0.38	644.46	644.93	1.19	644.95			-1.14	-0.65
13003 Viento Del Sur	646.35							644.97	0.42	644.98	645.43	1.39	645.46			-1.37	-0.89
HC 13101 Viento Del Sur	646.81							646.03	0.21	646.03	646.55	1.01	646.57			-0.78	-0.24
13106 Viento Del Sur	649.11										647.08	0.79					
HC 13107 Viento Del Sur	647.56	645.56	0.14	645.56				646.41	1.08	646.43	647.02	1.48	647.05	-2.00		-1.13	-0.51
13109 Viento Del Sur	648.47	645.57	0.36	645.57				646.62	1.80	646.67	647.23	2.29	647.31	-2.90		-1.80	-1.16
13202 Viento Del Sur	648.41	646.46	1.49	646.49				647.90	4.25	648.18	648.46	4.93	648.84	-1.92		-0.23	0.43
13202 Viento Del Sur	648.41	646.36	2.16	646.44				648.86	6.05	649.43	649.59	7.70	650.51	-1.97		1.02	2.10
HC 13203 Viento Del Sur	648.99	645.20	1.69	645.24	645.20	1.72	645.24	647.28	4.37	647.58	647.91	5.09	648.31	-3.75	-3.75	-1.41	-0.68
13205 Viento Del Sur	649.74				646.11	1.87	646.16	647.71	3.59	647.91	648.32	4.54	648.64		-3.58	-1.83	-1.10
502 Whitetail	650.37	646.19	0.29	646.19	646.69	1.34	646.72	647.73	2.68	647.84	648.30	3.49	648.48	-4.18	-3.65	-2.53	-1.89
HC 13208 Viento Del Sur	649.51				647.98	1.83	648.03	648.73	1.47	648.76	649.21	2.00	649.27		-1.48	-0.75	-0.24
507 Whitetail	649.37				649.22	1.74	649.27	648.45	0.81	648.46	648.85	1.21	648.88		-0.10	-0.91	-0.49
500 Whitetail	644.78	644.29	0.20	644.29	645.86	3.35	646.03	646.19	2.27	646.27	646.91	2.88	647.04	-0.49	1.25	1.49	2.26
Front door 412 Whitetail	648.13	644.00	0.16	644.00	644.84	2.13	644.91	645.51	2.66	645.62	646.19	3.49	646.38	-4.13	-3.22	-2.51	-1.75
408 Whitetail	646.52	642.06	1.83	642.11	642.63	2.52	642.72	643.83	3.89	644.06	644.57	4.42	644.87	-4.41	-3.80	-2.46	-1.65
404 Whitetail	643.85							643.70	1.10	643.72	644.50	1.78	644.55			-0.13	0.70
407 Whitetail	644.90				641.20	1.43	641.23	642.71	2.68	642.82	643.56	3.38	643.73		-3.67	-2.08	-1.17
314 Horsethief Tr.	642.90	639.13	1.34	639.15	639.80	2.09	639.87	641.07	3.70	641.28	641.79	4.49	642.10	-3.75	-3.03	-1.62	-0.80
Front door 312 Horsethief Tr.	643.10				639.43	0.66	639.43	640.51	2.78	640.63	641.16	3.64	641.36		-3.67	-2.47	-1.74
													Average=	**-2.95**	**-2.61**	**-1.14**	**-0.47**

*Computational Cells with no data did not show structure being inundated in hydraulic scenario

Table 7 – Depth Comparison from Max Breach Scenario and Survey Report from Mr.O'Hara

Address	Survey Report of Depth Inside Structure (ft)	Hydraulic Model Results of 6 hr Breach (Qp=7,169)			Difference (Reported-Computed)
		Depth (ft)	Velocity (ft/s)	Total Energy (ft)	
314 Horsethief Tr.	3.00	3.02	3.85	3.25	0.25
13003 Viento Del Sur	0.42	0.44	1.30	0.46	0.04
13106 Viento Del Sur	0.25	0.31	0.40	0.31	0.06
13109 Viento Del Sur	No Depth Provided	1.78	1.55	1.81	
13200 Viento Del Sur inside	1.50	2.37	3.01	2.51	1.01
13200 Viento Del Sur outside	3.00	2.51	3.02	2.65	-0.35
13201 Viento Del Sur	No Depth Provided	3.40	3.27	3.56	
13202 Viento Del Sur	0.83	3.03	4.47	3.35	2.51
13205 Viento Del Sur	3.00	2.81	4.09	3.07	0.07
404 Whitetail	No Depth Provided	1.51	1.94	1.57	
407 Whitetail	2.83	2.40	2.49	2.50	-0.34
408 Whitetail	3.75	3.61	3.51	3.80	0.05
500 Whitetail	No Contact Made	2.66	2.77	2.77	
502 Whitetail	3.00	1.70	3.07	1.85	-1.15
507 Whitetail	0.67	0.62	1.00	0.64	-0.03
Front door 312 Horsethief Tr.	1.67	1.22	2.32	1.31	-0.36
Front door 412 Whitetail	3.25	2.12	3.24	2.28	-0.97
HC 13101 Viento Del Sur	0.67	0.44	1.11	0.45	-0.21
HC 13107 Viento Del Sur	1.08	1.27	0.94	1.29	0.20
HC 13203 Viento Del Sur	3.50	3.62	4.71	3.96	0.46
HC 13208 Viento Del Sur	0.75	0.82	1.75	0.86	0.11
HC 600 Bear Canyon	0.25	0.42	1.54	0.46	0.21
near front door 13101 Viento Del Sur	1.33	0.51	0.78	0.52	-0.81
Watermark 601 Bear Canyon	0.33	0.60	1.20	0.62	0.29
				Average	**0.05**

As shown in the tables above (see rightmost column of Tables 5 and 6), most of the computed water surface elevations were lower than the reported water surface elevations which were reported by the residents and measured by Mr. O'Hara. The closest matching water surface elevations were from the maximum breach peak flow of 7,169 cfs. This peak flow was also compared to the depths inside the houses, which are presented in the text of Mr. O'Hara's report. Table 7 shows that the model results inside of each structure compare well with the depths reported inside of each house.

This underestimation of the water surface elevations could imply the following:

1. This analysis is underestimating the rate and how much water went through and/or over the embankment. This may be due to: 1) an estimation of the influence of the 4 rain gages was required, whereas ideally, a rain gage was located at the centroid of the watershed and rainfall was measured there, and 2) the assumption of the rainfall time distribution is based upon regional analysis and may greatly differ from the actual distribution.
2. The complicated effects of wave and velocity run-up actions may account for higher water surface recordings on the outside of a structure than what is computed in the hydraulic model. Note that the 2D models cannot account for these phenomena.
3. Both the model and the survey are in the North American Vertical Datum (NAVD 88) and the North American Datum (NAD 83) for Central Texas. However, the depth values from the model match more closely to the reported values within the structures than the surveyed values outside of the structures. This is likely because the surface DEM developed from the 2' contours differs slightly at structures, as the elevation of each structure is removed from the contour data. The HEC-RAS model used a higher roughness value to account for structure locations.

Large differences from the model and the surveyed high-water marks are most likely due to wave run-up and velocity head turning into potential energy as the moving water hits against the upstream side of the structures. In fact, water surface measurements from one of the houses resulted in the upstream measurement being higher than the downstream measurements by approximately 2 feet as reported by Mr. O'Hara. For example, a water velocity of 10 feet per second at a water surface depth of 5 feet would produce a velocity "head" of 1.5 feet of energy. This means that if the water is in a house where the water is not flowing, the high-water mark would be at 6.5 feet deep (5 + 1.5 feet) whereas the flow immediately outside the house would flow at a depth of 5 feet. The hydraulic model would show the 5 feet depth.

Wave Runup Analysis

To analyze the wave runup on houses, a computational fluid dynamic (CFD) model was developed. CFD models can analyze hydraulic performance in 3 dimensions (x, y, and z), which is significantly more complex when compared to the 1D or 2D models within HEC RAS. A Flow3D model was developed by putting the structure data from GIS into a 3-Dimensional model. The flood surge from the breach was then input upstream of the structures. Figure 12 presents a section view of the surge behavior on a structure before the main flood surge concentrates in the neighborhood. Both images show how complex flow depths around a structure are and computed results show significantly higher depths due to the ability for the CFD model to analyze the vertical momentum of the wave runup. The CFD model[8] is also presented to highlight the difficulty in predicting an exact depth and velocity at each structure within a 1D or 2D model and shows that the runup can affect the measured water surface

[8] https://www.flow3d.com/

elevation. The hydraulic models' inability to simulate this momentum runup as well as the issues related to the velocity head were the main reasons that the models under-predicted the measured water surface elevations by an average of about 0.5 feet. However, the differences between scenarios are of the most interest so the analyses presented are deemed valid.

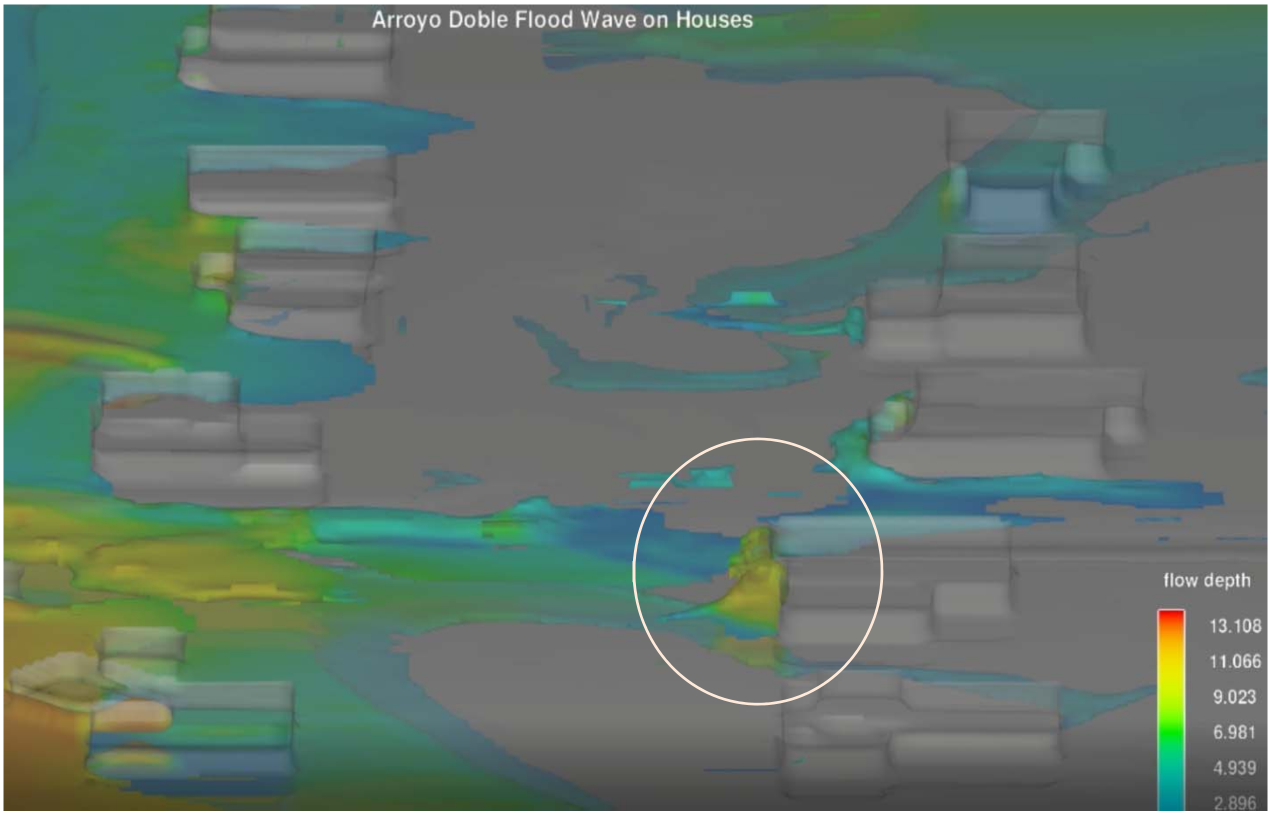


Figure 12 - CFD model showing initial wave runup onto structure before the full peak concentrates in the Neighborhood

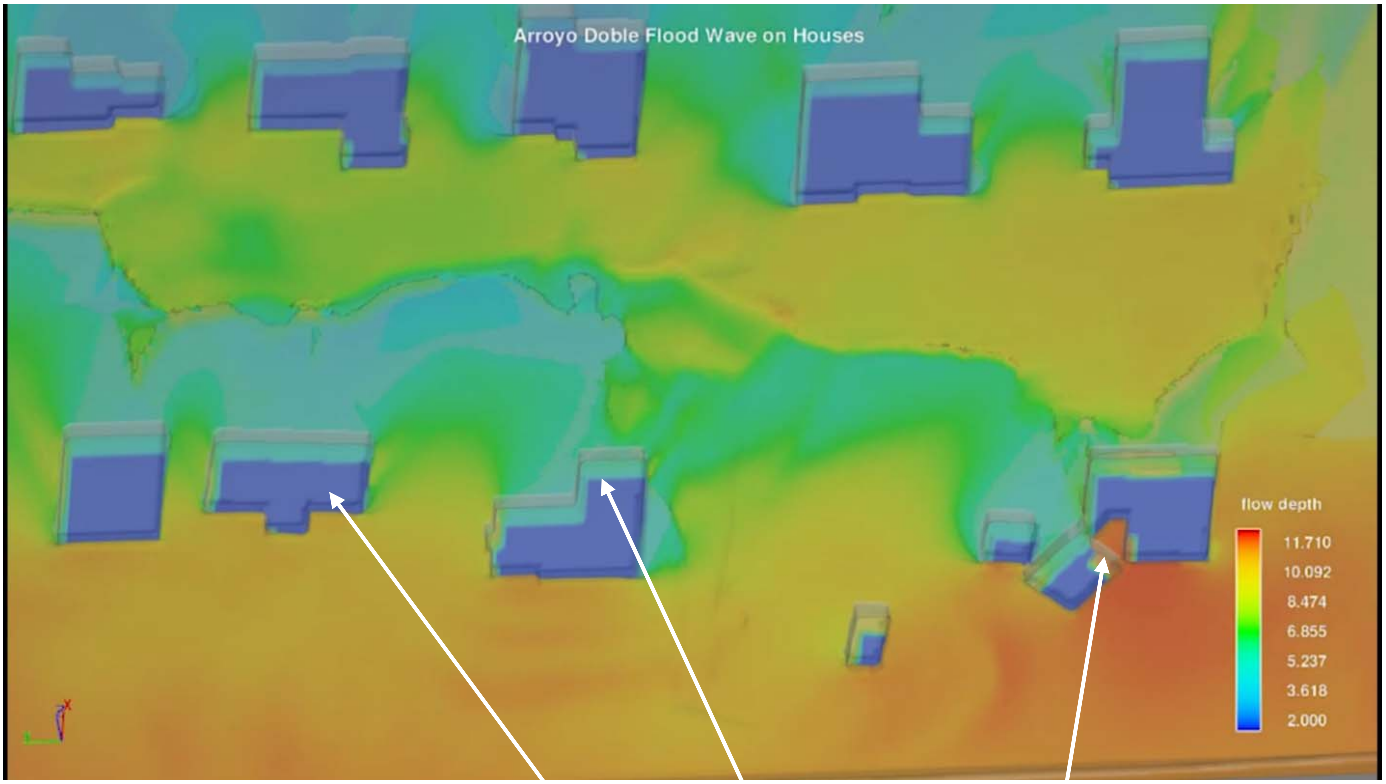


Figure 13 – Depth of Flood Wave Runup on Houses, developed within Flo3D CFD Software. Google earth image presents arrows to houses in model for a reference to location.

Summary of Results

The purpose of this analysis was to understand the failure modes of the embankment and the effects on the Arroyo Doble neighborhood. Hydrologically, it appears that the amount of runoff and the amount and rate of water that was produced when the embankment failed was on the higher end of our hydrologic estimates. Additionally, the hydraulic phenomena of momentum runup and velocity head may have cause some of the scenarios in the hydraulic models to under-predict the measured water surface elevations by an average of 0.5 feet. However, the concentration of this discussion is on the relative differences of three scenarios, so the results and analyses presented are deemed valid.

Based on the hydrologic and hydraulic modeling, it is estimated if the embankment would have not failed, but instead simply overtopped and stayed, or had a culvert placed, the maximum released flow for this event would have been equal to or less than the inflow to the UPRR embankment, which was between 900 and 1,300 cfs depending on the storm distribution. This would have been much lower that our predicted peak flow rates of what actually happened in October 2015. The figures below present four maps, the first two are the result of the water surface elevations and total energy on the structures from the breach estimates, the third presents the impact on structures if the embankment did not breach but simply overtopped, and the fourth presents the impact on structures if the railroad embankment did not exist. As shown, the number of houses inundated is fewer and the flooding depth is less for the ones that are inundated. Had the UPRR Railroad corrected this drainage deficiency in the past, the amount of damage caused by this failure could have been greatly diminished.

The following are the conclusion as a result of the study in regard to the October 2015 flood event.

1. If the railroad embankment had not existed, some of the residences would still have been flood, but the number of residences flooded, and the depth of the flooding would have been much lower.

2. If the railroad embankment was in existence but simply overtopped and did not breach, the flooding extents and water depths would have been greater than if the railroad embankment did not exist. This is because the existing embankment diverts flow that would normally drained to Onion Creek but was diverted north along the railroad embankment and towards Bear Creek.

3. For the condition of the railroad embankment in existence and a breach of the embankment, which is the situation that occurred on October 2015, the flooding extent and water depths were greater than if the railroad embankment was not there. In addition, the October 2015 embankment breach resulted in greater flooding extents and water depths than the condition of the existence of the railroad embankment with overtopping and no embankment breach.

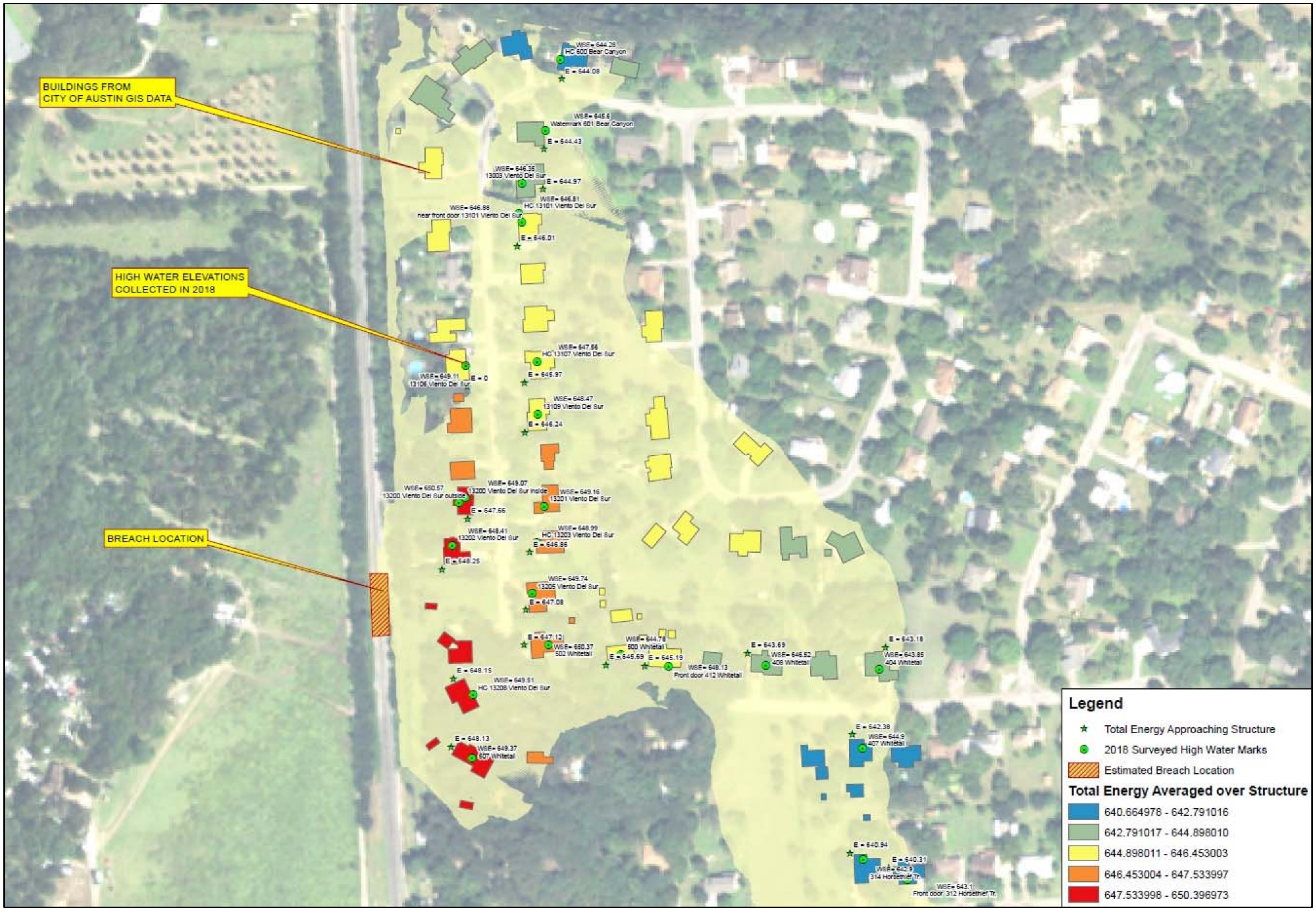


Figure 14 – WSE and Energy Estimates from the 2D Unsteady Breach Model (Qp=4,637)

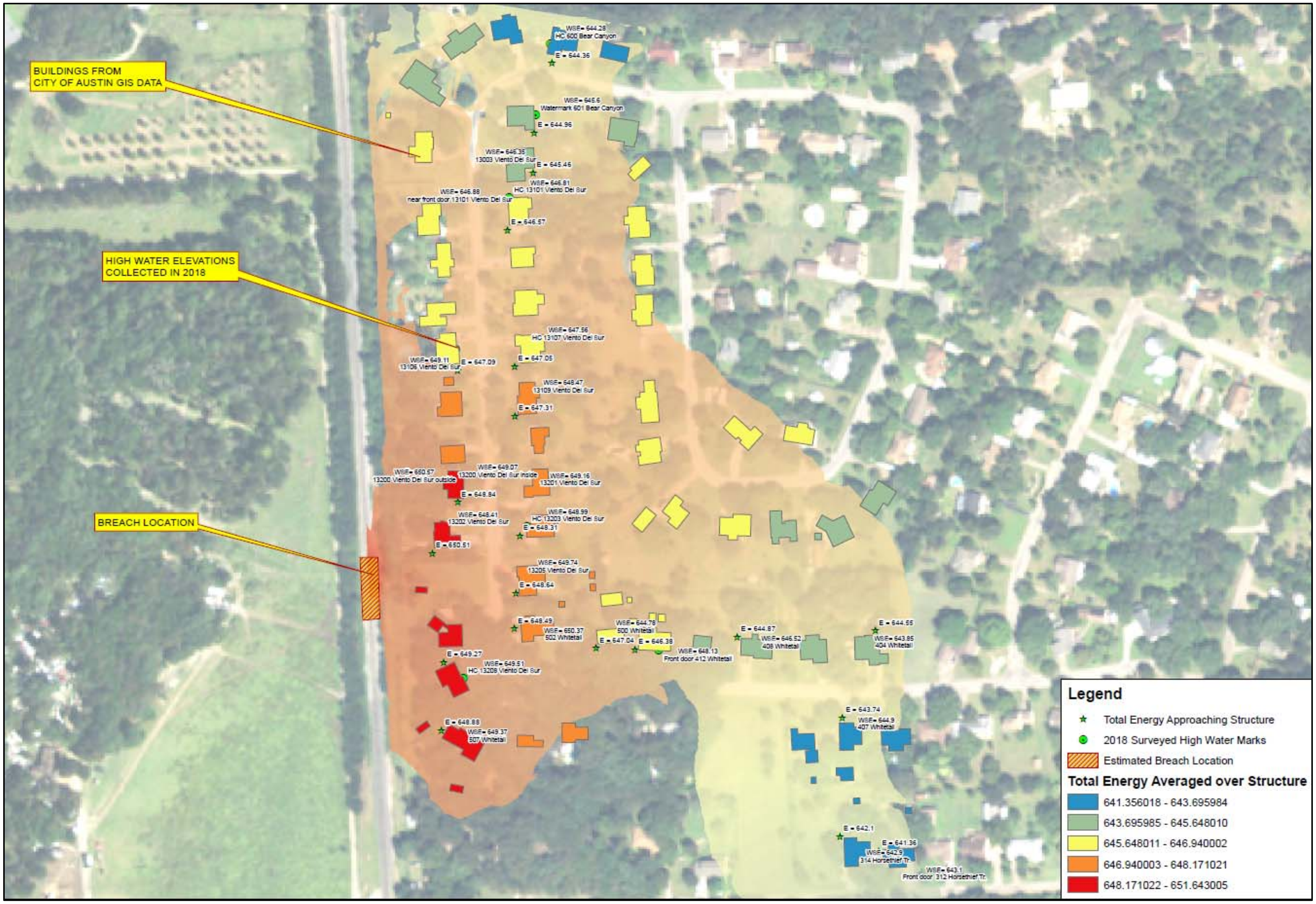


Figure 15 – WSE and Energy Estimates from the 2D Unsteady Breach Model (Qp=7,169)

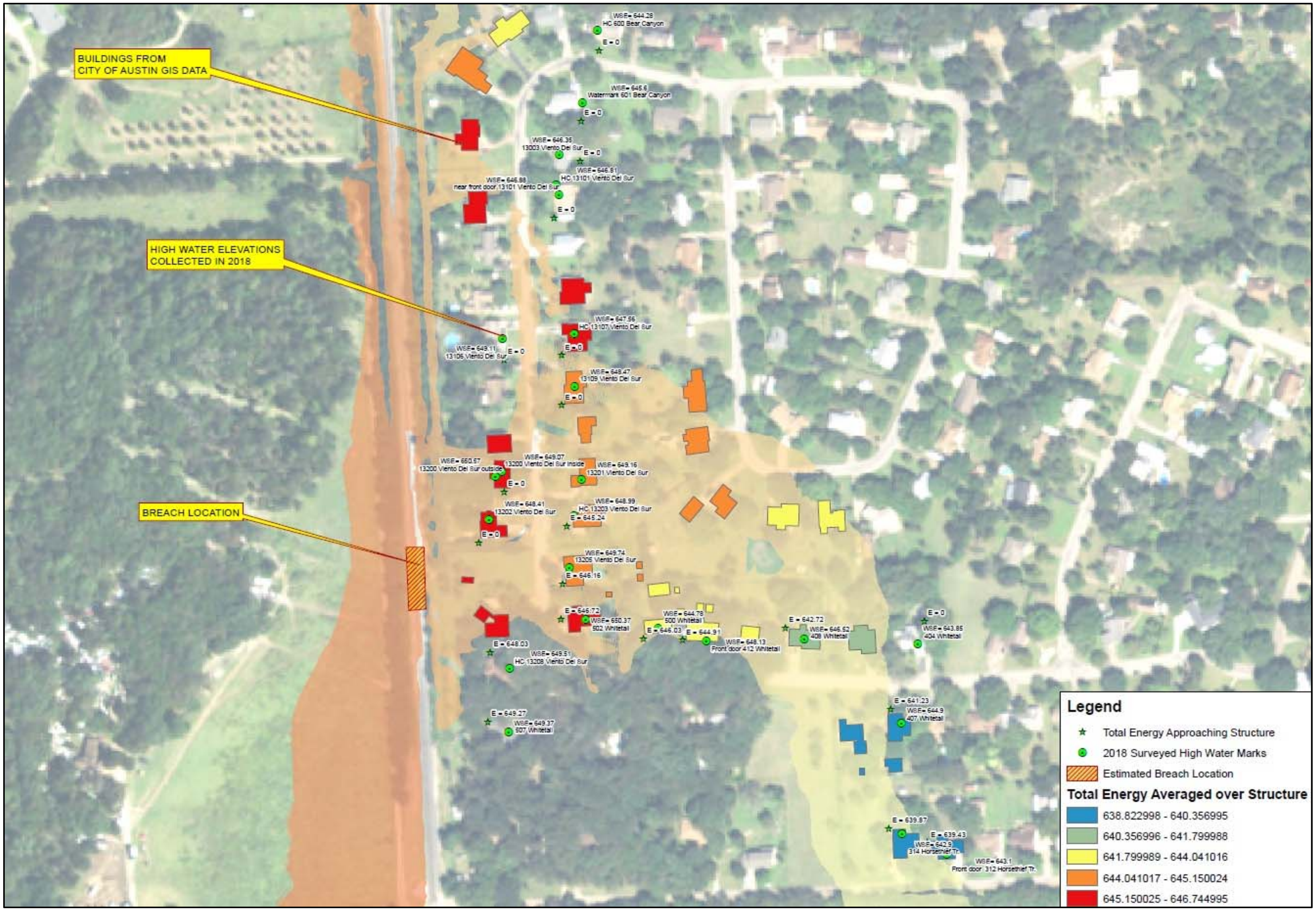


Figure 16 - WSE and Energy Estimates from the 2D Unsteady Existing Model, which assumes the UPRR Embankment does NOT Fail

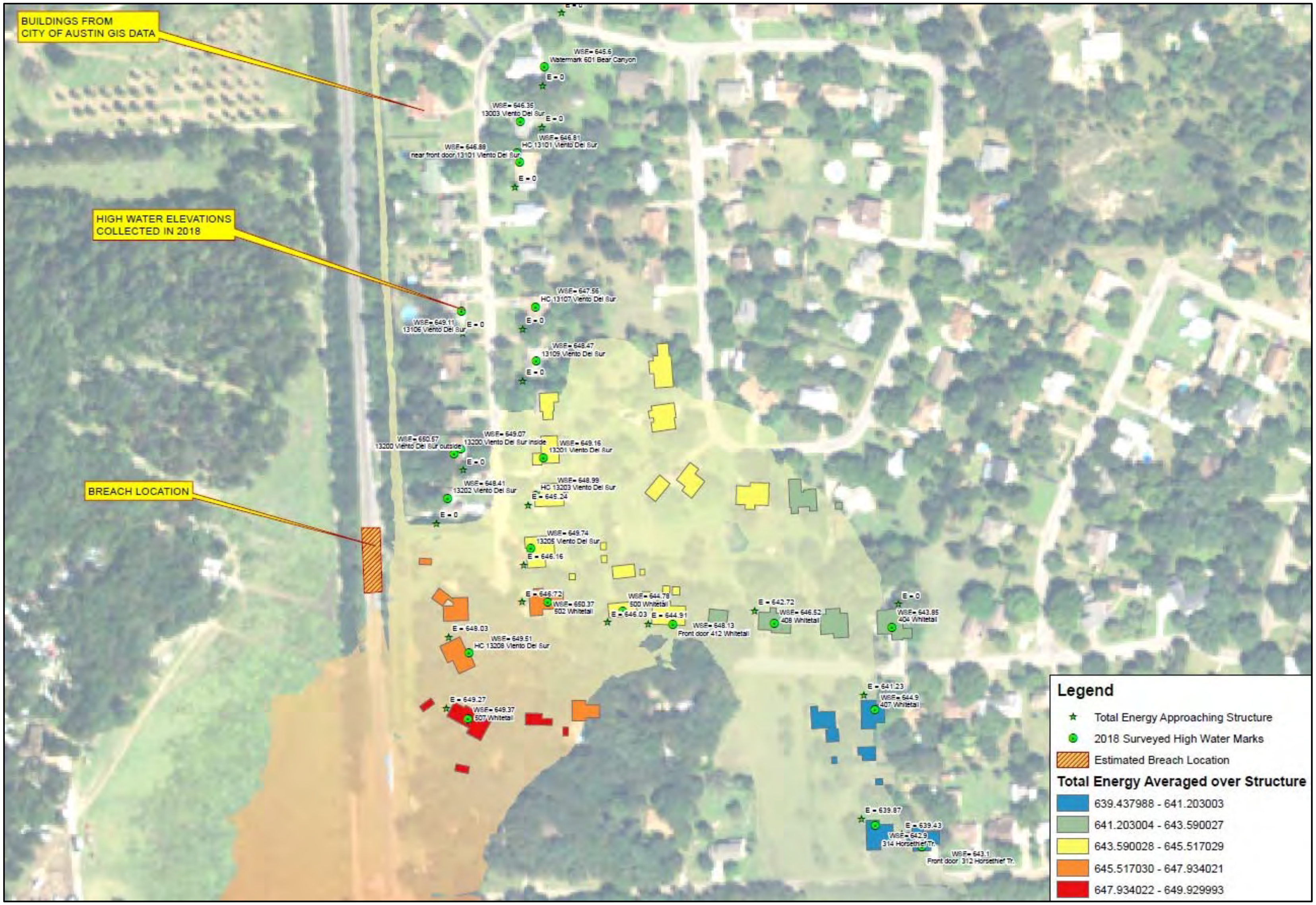


Figure 17 - WSE and Energy Estimates from the 2D Unsteady no-track Model, which assumes the UPRR embankment does not exist

IV. Review of Maintenance Activity and Standards for Railroad Work

Based on review of aerial imagery (see Figure 18), it appears that this location has been damaged and there have been maintenance issue from rainfall events in the past. During an event in October of 2013, there was an approximately 20-year rainfall event in the same drainage basin. Based on review of Google Earth imagery, it appears that a breach in the same location had occurred in October 2013 and was subsequently repaired by UPRR.

According to the UPRR Engineering Track Maintenance Handbook (Revised in 2015) the general considerations for drainage are to design for the maximum expected runoff from rain, melting snow, or other sources. Additionally, according to the UPRR Track Maintenance Manual, Section 1.3.3 note 4, the guidance states:

When a washout occurs during heavy runoff, determine whether existing devices are obstructed.

- *If they are, remove the obstructions.*

or

- *If they are not, install additional or larger culverts.*

After the 2013 event, or possibly previous flood events, according to the UPRR Track Maintenance manual, UPRR should have installed a culvert or a series of culverts or possibly other drainage measures. Note that the 2015 event was a large event with an annual exceedance probability (AEP) of 0.4%, which correlates to a 1 in 250-year event. The two events in October that appeared to cause damage were smaller one in 2013, with an AEP of 5% (1 in 20 years). Had UPRR corrected the known drainage problems, it is possible that the 2015 flood event would not have breached the embankment and the damages to the neighborhood could have been significantly diminished.

Table 7 presents the recorded rainfall depths and frequencies of known time periods events that have caused damage to the UPRR embankment near the Arroyo Doble subdivision.

Table 8 – Rainfall Depths and Frequencies that have caused Erosion or Failure at the UPRR Embankment

Storm Event	24 Hour Rainfall Depth Inverse Distance Weighted (IDW) Results (inches)	Approximate Storm Frequency (Years)	Annual Exceedance Probability (AEP)
Oct 30 2015	12.1	250	0.4%
Oct 10 2013	6.01	10	10.0%
Oct 31 2013	7.18	20	5.3%

The figure below presents the Google Earth imagery from 2013 and the most recent imagery of 2018, showing the extent of repairs made after the 2015 flood event.

Figure 18 – Images of Track Repairs in 2013 and after the 2015 Event

V. What Could Have Been Done with Drainage

Topography draining to the site shows that most of the 200 acres in the drainage area drains to the UPRR track, South of Horsethief Trail Road. The drainage area directly upstream of the breach location is 31.4 acres and drains directly towards an existing drainage ditch downstream of the embankment. Due to the low-lying topography, a series of cross culverts would be required to catch as much drainage as possible before it reaches the breach location. First, it may be possible to design a series of smaller cross culverts upstream of Horsethief Trail Road. A drainage ditch would be required to convey flows from the UPRR track eastward to Onion Creek. Secondly, a series of culverts could be installed at the breach location that would drain flows into the existing drainage ditch downstream of the breach location, and lastly, an overflow ditch or channel can be graded to take high flows to Bear Creek before ponding occurs and overtopping the embankment.

Figure 19 – Concept Level Plan of a potential Drainage Solution

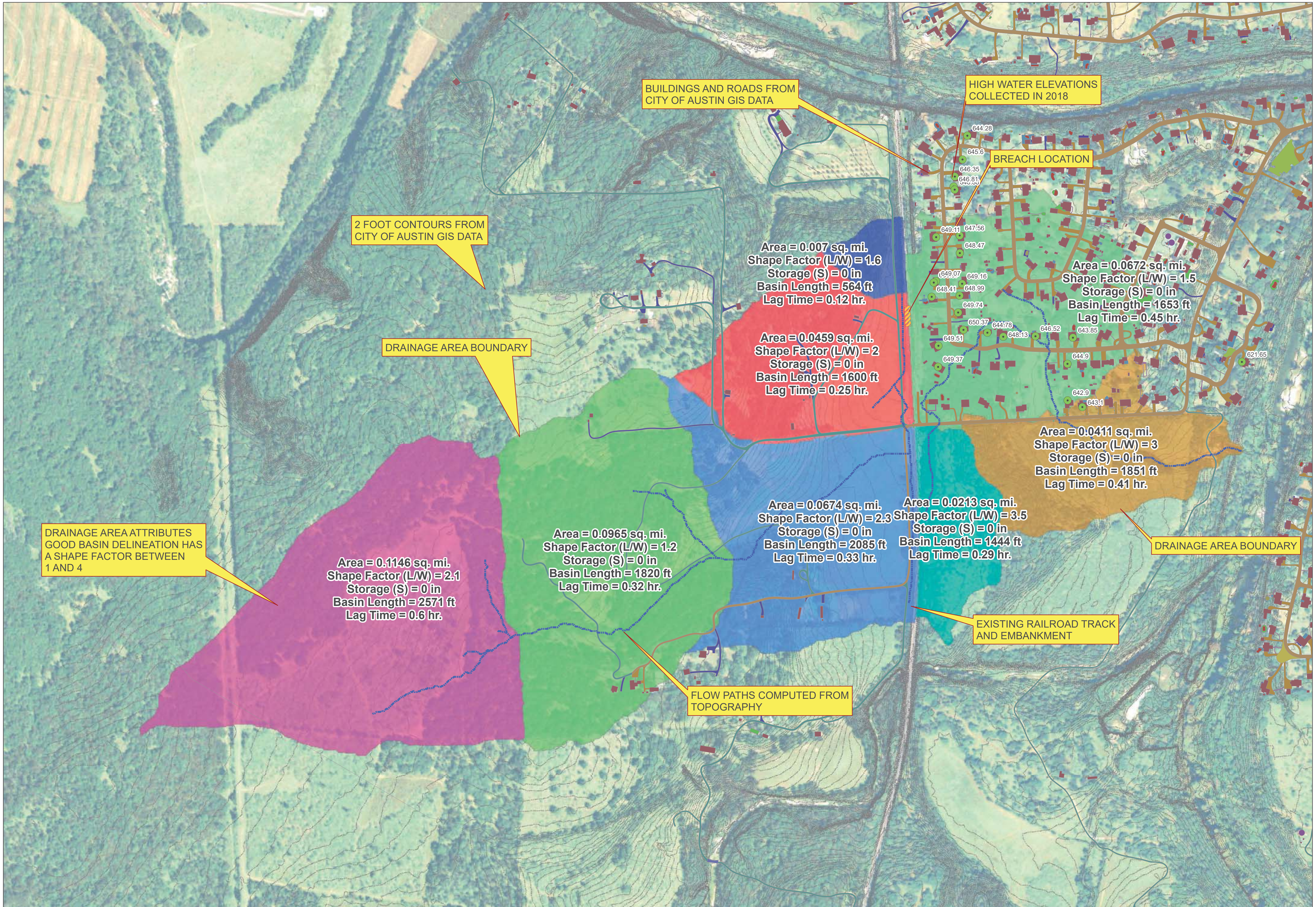
The natural drainage paths derived from the City of Austin's GIS contours take water through the neighborhood. A cursory review of the information indicates that water could be safely transported from upstream of the UPRR embankment to Onion

Creek and Bear Creek, although it would require a significant amount of work outside of UPRR's Right of Way (ROW).

VI. Conclusions on Maintenance Manual and Drainage

The drainage to this site is approximately 200 acres and warrants a proper drainage study and drainage pathway to reduce the likelihood of the railroad embankment failing again in the future. If the railroad had established a culvert and drainage path before or even after the community was built, it is possible that some of the property damage could have been diminished and perhaps some not damaged at all, even for a large storm event such as the one that occurred in October of 2015. There appears to have been a failure in the past from an event that was significantly smaller. According to UPRR Engineering and Maintenance Manual, this failure should have prompted some type of corrective action to repair the drainage deficiencies. If an entity wants to design and construct a more robust drainage solution, it appears that this could be possible; however, much of the work required for a drainage solution would be outside of the UPRR ROW. UPRR could install a series of cross culverts to reduce the likelihood of an embankment failure occurring in the future.

Attachment A – GIS Maps



DTW AND ASSOCIATES

SHEET TITLE:

ARROYO DOBLE NEIGHBORHOOD MAP SHOWING DRAINAGE AREA AND FLOW PATHS DERIVED FROM CITY OF AUSTIN GIS DATA

DESIGNED: NA	CHECKED:
DRAWN: Gerald E Blackler, PE, PhD	DATE: JULY 2018
PROJECT NUMBER:	
DESIGN PACKAGE NUMBER:	

ENGINEER CERTIFICATION:

THIS MAP IS DRAFT AND CONFIDENTIAL
THIS MAP IS ONLY INTENDED FOR VIEWING BY INDIVIDUALS IT HAS SPECIFICALLY BEEN ADDRESSED

CONTOUR INTERVAL: 2-FOOT

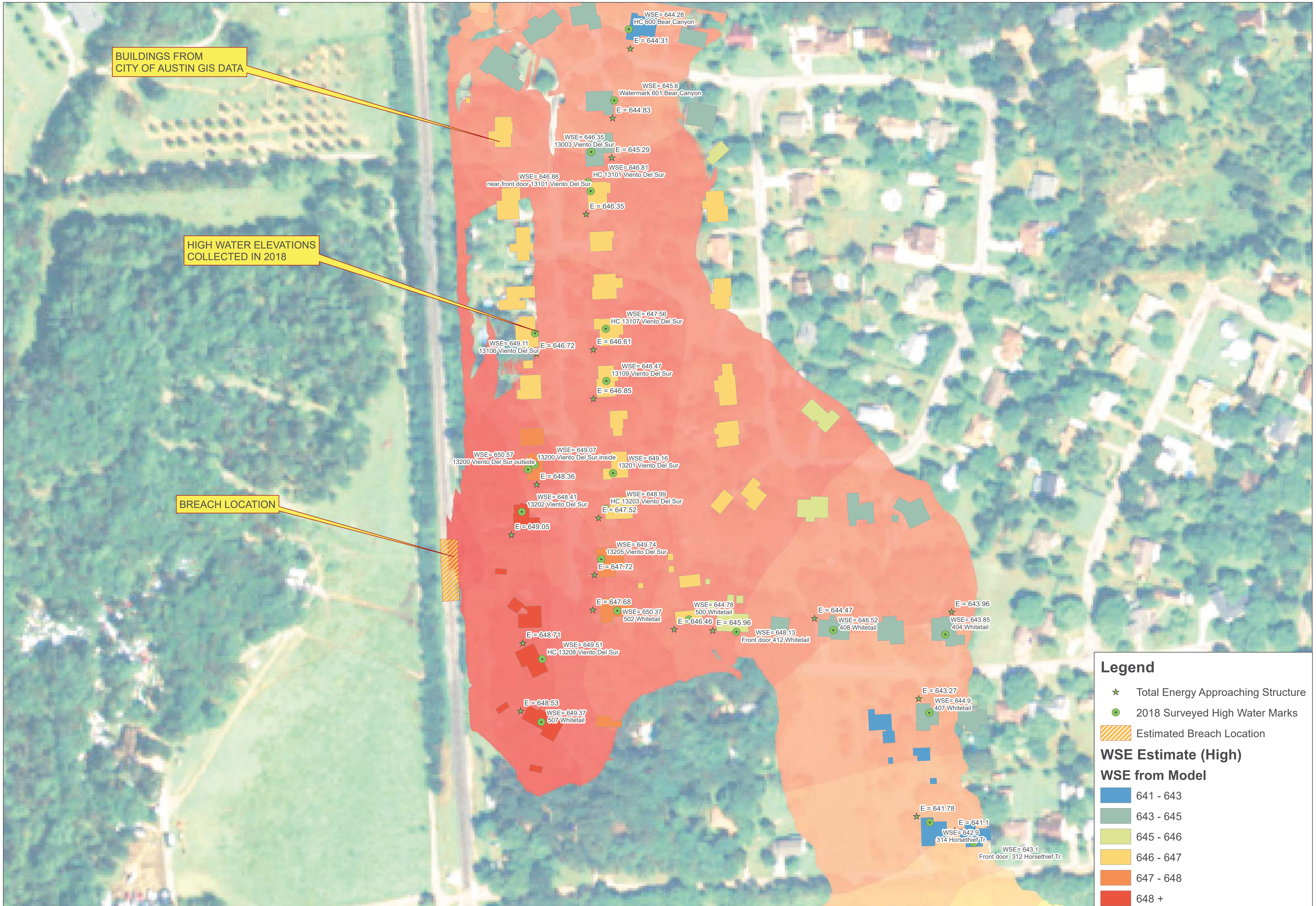
DATUM:
AD_1983_STATEPLANE_TEXAS_CENTRAL_FIPS_4203_FEET
PLANE COORD. - NORTH, VERTICAL - NAVD88

SHEET NUMBER:

1 OF 7

ORIGINAL SCALE:

1 inch = 300 feet



DTW
AND
ASSOCIATES

SHEET TITLE:
ARROYO DOBLE NEIGHBORHOOD MAP SHOWING MEASURED AND COMPUTED WSE VALUES FOR THE STEADY FLOW RATE ESTIMATE Q=3,996 CFS

DESIGNED: NA
CHECKED:
DRAWN: Gerald E Blackler, PE, PhD
DATE: JULY 2018
PROJECT NUMBER:
DESIGN PACKAGE NUMBER:
ENGINEER CERTIFICATION:

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ADDRESSED

CONTOUR INTERVAL: 2-FOOT

DATUM:
AD_1983_STATEPLANE_TEXAS_CENTRAL_FIPS_4203_FEET
PLANE COORD. - NORTH, VERTICAL - NAVD88

SHEET NUMBER:
2 OF 7

ORIGINAL SCALE:
1 inch = 100 feet

DTW
AND
ASSOCIATES

SHEET TITLE:

ARROYO DOBLE NEIGHBORHOOD
MAP SHOWING MEASURED AND
COMPUTED WSE VALUES FOR
THE 24 HR UNSTEADY BREACH
ESTIMATE QP=4,637 CFS

DESIGNED: NA	CHECKED:
DRAWN: Gerald E Blackler, PE, PhD	DATE: JULY 2018
PROJECT NUMBER:	
DESIGN PACKAGE NUMBER:	

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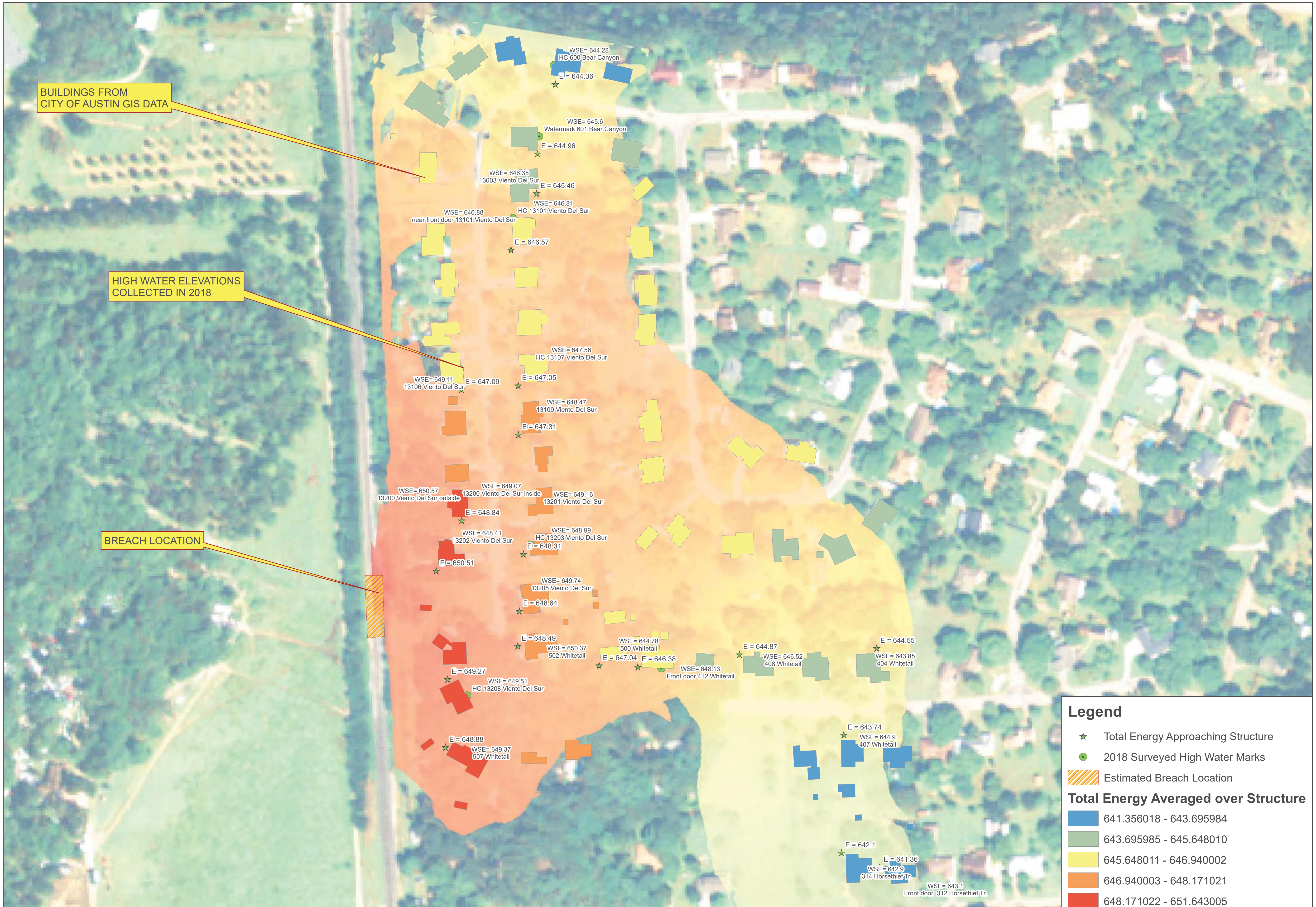
PLANE COORD. - NORTH, VERTICAL - NAVD88

SHEET NUMBER:

3 OF 7

ORIGINAL SCALE:

1 inch = 100 feet



DTW AND ASSOCIATES

SHEET TITLE:

ARROYO DOBLE NEIGHBORHOOD MAP SHOWING MEASURED AND COMPUTED WSE VALUES FOR THE UNSTEADY 6 HR BREACH ESTIMATE QP=7,169 CFS

DESIGNED: NA	CHECKED:
DRAWN: Gerald E Blackler, PE, PhD	DATE: JULY 2018
PROJECT NUMBER:	
DESIGN PACKAGE NUMBER:	

ENGINEER CERTIFICATION:

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CONTOUR INTERVAL: 2-FOOT

DATUM:
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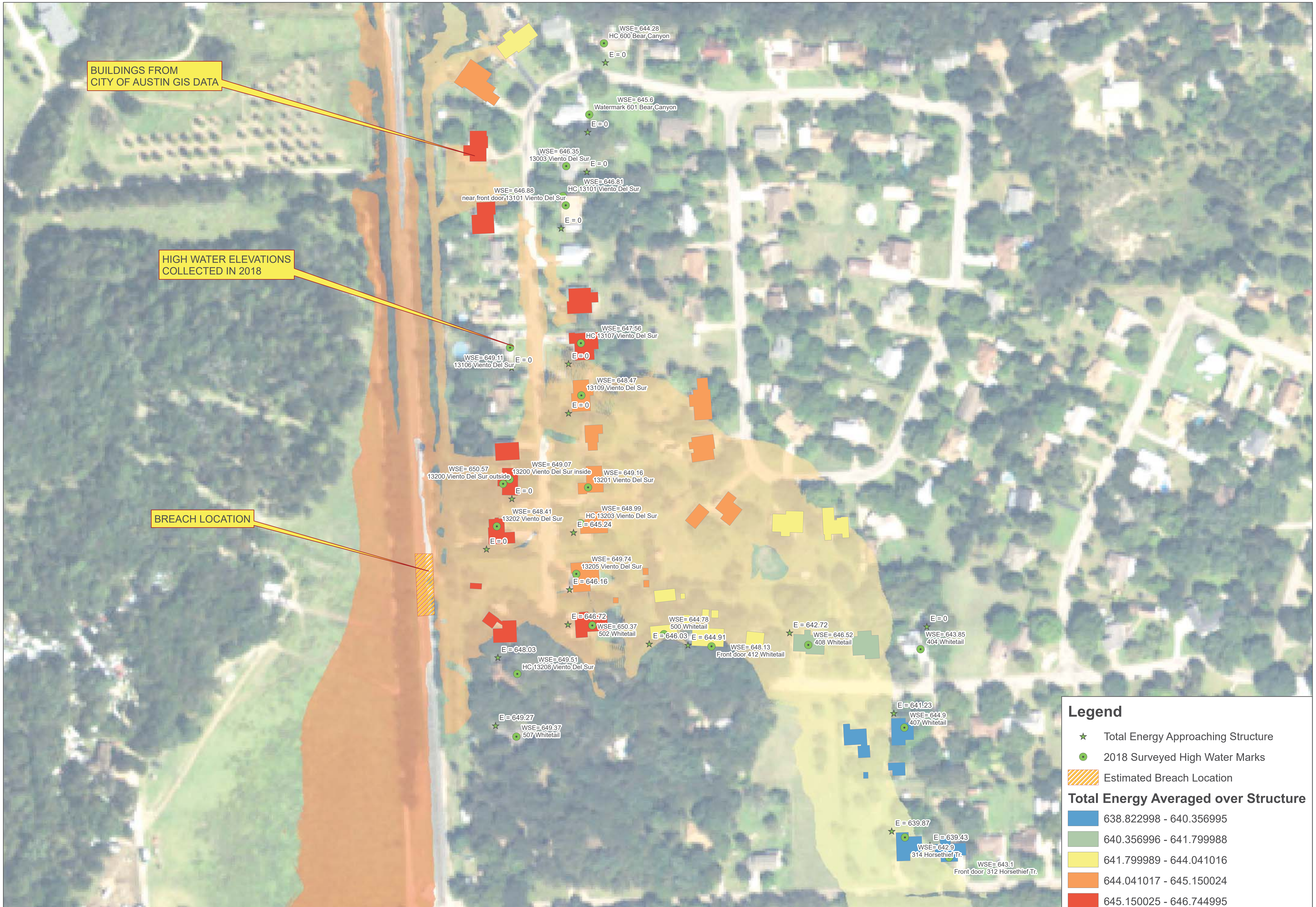
PLANE COORD. - NORTH, VERTICAL - NAVD88

SHEET NUMBER:

4 OF 7

ORIGINAL SCALE:

1 inch = 100 feet



DTW
AND
ASSOCIATES

SHEET TITLE:

ARROYO DOBLE NEIGHBORHOOD
MAP SHOWING MEASURED AND
COMPUTED WSE VALUES
ASSUMING THE EMBANKMENT
DOES NOT BREACH

DESIGNED: NA
CHECKED:
DRAWN: Gerald E Blackler, PE, PhD
DATE: JULY 2018
PROJECT NUMBER:
DESIGN PACKAGE NUMBER:

ENGINEER CERTIFICATION:

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BY INDIVIDUALS IT HAS SPECIFICALLY BEEN
ADDRESSED

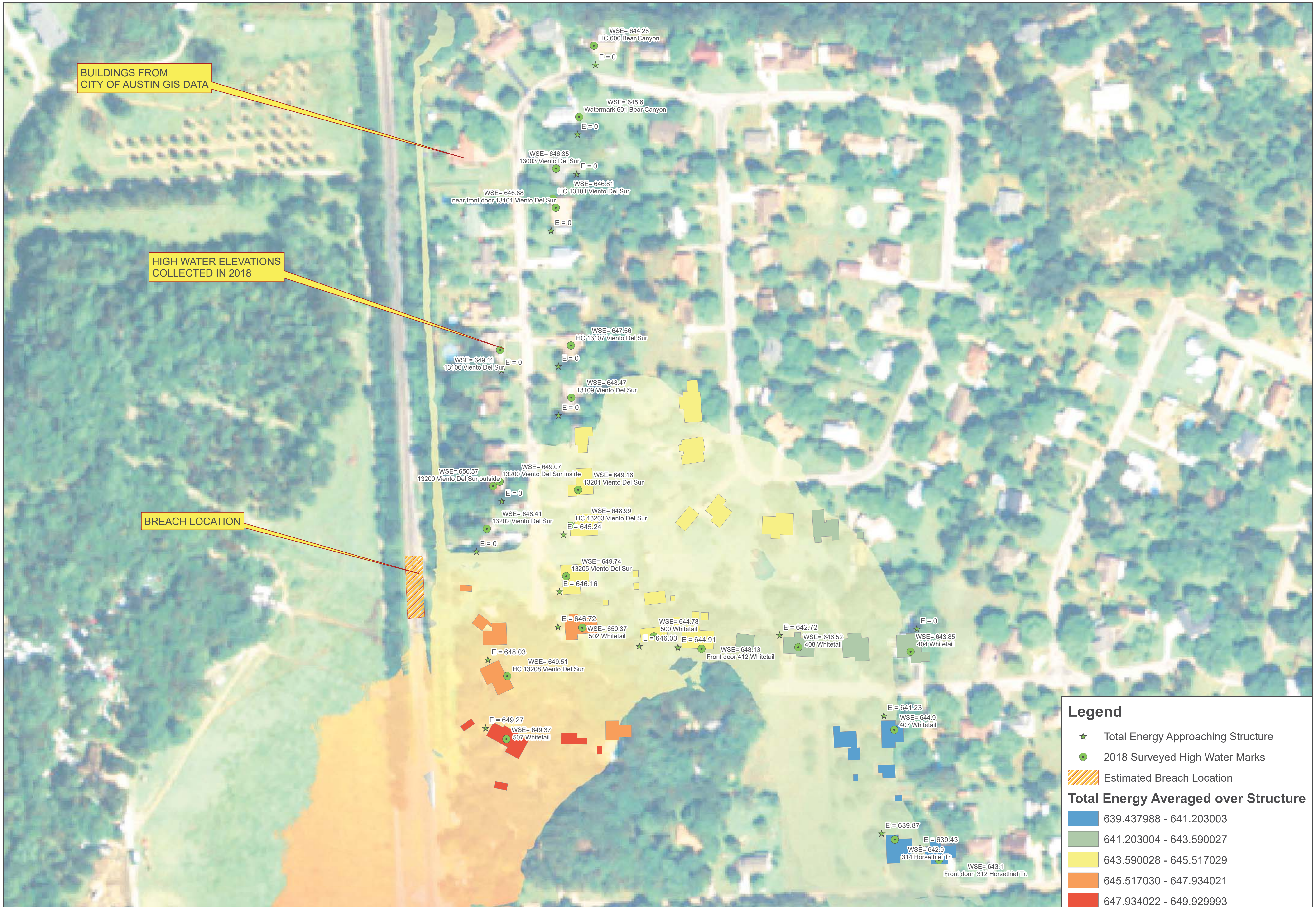
CONTOUR INTERVAL: 2-FOOT

DATUM:
AD_1983_STATEPLANE_TEXAS_CENTRAL_FIPS_4203_FEET
PLANE COORD. - NORTH, VERTICAL - NAVD88

SHEET NUMBER:

5 OF 7

ORIGINAL SCALE:
1 inch = 100 feet



DTW AND ASSOCIATES

SHEET TITLE:

ARROYO DOBLE NEIGHBORHOOD MAP SHOWING MEASURED AND COMPUTED WSE VALUES ASSUMING THE EMBANKMENT DOES NOT EXIST

DESIGNED: NA	CHECKED:
DRAWN: Gerald E Blackler, PE, PhD	DATE: JULY 2018
PROJECT NUMBER:	
DESIGN PACKAGE NUMBER:	

ENGINEER CERTIFICATION:

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CONTOUR INTERVAL: 2-FOOT

DATUM:
AD_1983_STATEPLANE_TEXAS_CENTRAL_FIPS_4203_FEET
PLANE COORD. - NORTH, VERTICAL - NAVD88

SHEET NUMBER:

6 OF 7

ORIGINAL SCALE:
1 inch = 100 feet